Physical Model Studies For the Bluestone Lake Dam Safety Assurance Project Design

Kenneth C. Halstead[1] and Sean L. Smith[2]

Abstract

The Corps of Engineers Dam Safety Assurance (DSA) program provides for upgrading existing dams which have hydrologic or seismic deficiencies to allow them to function safely, and as originally intended. The Bluestone Dam has no seismic deficiencies. However, the project does have hydrologic deficiencies. Based on present day design criteria, the Probable Maximum Flood (PMF) would exceed the top of the existing dam by 20.8 feet, assuming the dam to be raised in place. In addition, geotechnical and structural stability computations demonstrate that the dam is in imminent danger of failure at flood pool levels approaching the top of the existing dam (elevation 1535), or approximately 70% of the PMF. These findings raise serious concerns for public safety because a dam failure would put more than 115,000 persons at risk and cause over $6.5 billion in property damage. Therefore, Bluestone Dam must be upgraded to meet present day design criteria provided by ER 1110-2-1155.

Major features of the recommended plan for correction of the hydrologic deficiencies include raising the top of the dam 8 feet to elevation 1543.0, stabilizing the existing dam monoliths with high strength, multi-strand steel anchors, and the modification of the six inactive hydropower penstocks to provide additional discharge capacity. During the design phase for the Detailed Design Report (DDR), two physical models were employed to obtain a variety of design parameters and to verify the performance of the project with the recommended plan in place. This paper will present and discuss those physical model studies.

Bluestone Lake Project Location, History and Features

The Bluestone Lake project is located in southern West Virginia within the New River Basin which is a sub-basin of the Kanawha River Basin. The Bluestone Dam is situated across the New River near the town of Hinton, approximately 4500 feet (0.85 miles) upstream of the confluence with the Greenbrier River. Total reservoir capacity is 631,000 acre feet at the maximum flood control pool, elevation 1520 feet, which is equivalent to approximately 2.59 inches of runoff. The project began operation in 1949 and controls a 4604 square mile drainage area.

During the original authorization, planning, and design of Bluestone Lake, hydropower was a major project purpose. During the original construction, which began in 1942, six penstocks were built into a non-overflow section of the dam.

---

[1] Hydraulic Engineer, Water Resources Engineering Branch, Corps of Engineers, 502 Eighth St, Huntington, WV 25701-2070, (304) 528-7411, Kenneth.C.Halstead@lrh01.usace.army.mil
[2] Supervisory Hydraulic Engineer, Water Resources Engineering Branch, Corps of Engineers, 502 Eighth St, Huntington, WV 25701-2070, (304) 529-5523, Sean.L.Smith@lrh01.usace.army.mil

During a three year interruption in construction for World War II, extensive wartime electric power development caused hydropower at Bluestone to be reconsidered. Thus, in 1945, a decision was made to defer hydropower development, and the hydropower plant was never constructed.

The main body of the dam consists of the left and right abutment sections, the spillway section, assembly bay section, powerhouse intake section, and the non-overflow section. The dam is a straight, concrete gravity structure with a maximum height of 165 feet, and an overall top length of dam is 2,061.5 feet at elevation 1,535 feet. A gated concrete ogee spillway, located in the channel section of the dam, is 790 feet wide at a crest elevation of 1,490 feet. Spillway flow is controlled by 21 vertical slide gates, each being 31 feet high and 30 feet wide. The outlet works consist of 16 gated sluices through the lower portion of the spillway section. The sluices are 5'-8" wide by 10' high with an invert elevation of 1,389 feet, and are controlled by hydraulically operated gates from a gallery inside the dam. The six penstocks transition from 16' wide by 26' high rectangular section on the lake side to an 18' diameter circular section at the downstream toe of the dam. Hemispherical steel bulkheads are bolted in place near the upstream end of each penstock to provide closure. The downstream end of each penstock exits into a static pool of water at about tailwater level, in the area that was originally designated for a powerhouse.

Dam Safety Assurance Study

Since the Bluestone Dam was constructed and was placed in operation in 1949, over 50 years of additional hydrologic data has been collected and compiled. Accordingly, hydrologic design criteria for dams has also been revised and improved over that time period. In consideration of the number of dams that were designed and constructed in the early to mid 1900's, the Corps of Engineers initiated the Dam Safety Assurance (DSA) program. This program provides for the upgrading of existing dams that have hydrologic and/or seismic deficiencies, so that those structures can function safely, and operate as originally intended according to modern day design criteria and standards.

The original design for Bluestone Dam was based on the historical data of a 1916 hurricane that produced approximately 13 inches of rain. The present day design is based on the Probable Maximum Precipitation (PMP) of approximately 20 inches, as depth averaged. It is estimated that the Probable Maximum Flood (PMF), produced by the PMP, would rise above the top of the existing dam by 20.8 feet, assuming the top of dam to be raised and only the original discharge facilities available for releases.

The Huntington District of the U.S. Army Corps of Engineers completed the Dam Safety Assurance Evaluation Report for Bluestone Dam in May 1998. This study considered an array of alternatives to alleviate the hydrologic deficiency. The recommended plan for correction of the hydrologic deficiency would increase the maximum surcharge pool elevation from elevation 1523 to 1546.8. Prior to initial overtopping, this increase in maximum pool elevation will affect the stability of the dam at elevation 1532, or three feet below the existing top of the dam. Thus, the primary features of the recommended alternative are to anchor the dam, raise the non-overflow sections to prevent overtopping, and utilize the hydropower penstocks to provide additional discharge capacity for large flood events such as the PMF. As a part of the structural stability analysis, the “upper stilling basin”, located immediately downstream of the toe of the ogee spillway section, is being considered for its capability to provide some resistive forces against sliding and overturning for the spillway section of the dam. The existing spillway stilling basin was originally designed to contain and pass 430,000 cfs. With the recommended plan, the maximum discharge through the stilling basin will be more than doubled.

Currently, the Huntington District is involved in compiling and completing a Design Documentation Report (DDR) according to ER 1110-2-1150. The DDR will document the design details of proposed project features in support of the preparation of plans and specifications for Phase II of the DSA construction. The model study results will provide information to be used for structural stability computations, refinement of the hydrologic design flood routing, and will provide insight as to the potential for scour in the stilling basin. The model studies will be documented in the DDR.

Initial Physical Model Testing

The need for physical model tests of the proposed modifications to Bluestone Lake was identified in the DSA Program Evaluation Report for Bluestone Lake (Huntington District, 1998). The purpose of the initial model testing was to provide hydraulic design data for conditions expected to occur during a PMF that were beyond the scope of existing data and design guidance. The testing program was initiated by Huntington District in April 1999 during the Planning, Engineering, and Design scoping conference. Representatives from the District, Division, Headquarters and Waterways Experiment Station (WES, at that time, now CHL) attended the conference. The initial model testing program was accomplished in two phases. Phase A of the program included evaluation and measurement of parameters required for final design of the DSA modifications. In Phase B of the program, the dam’s performance was evaluated for allowing overtopping of monoliths 16 through 21 to provide additional discharge capacity.

The Corps’ Coastal and Hydraulics Laboratory (CHL) in Vicksburg, MS was consulted, and arrangements were made for their staff to conduct the necessary physical model studies. The model consisted of an undistorted linear 1:65 scale replication of the entire dam, spillway, outlet sluices and penstocks, along with a 2,200 foot reach of the downstream outlet channel and a 1000 foot reach of the pool upstream of the dam. Parameters evaluated during Phase A testing included extension of the existing discharge rating curves for the sluices and spillway. A rating curve for the proposed penstock modifications was also developed. Piezometric pressures were measured along the face of the spillway and within the stilling basin. Pressure measurements were also taken on the upstream face of a spillway gate and pier. A qualitative assessment of stilling basin performance was conducted and pressure data was collected for the downstream stilling weir. Discharges from the modified penstocks into the tailrace area were evaluated to determine the need for training walls and erosion protection in the vicinity of the east abutment.

The Phase B overtopping component of the model testing program consisted of the collection of pressure measurements on the dam crest and on top of a thrust block that is to be located at the downstream toe of the dam, while allowing overtopping of monoliths 16 through 21. This alternative proved to be very expensive, and would have negative impacts on a hydropower plant that is currently being investigated by another entity. Therefore, this alternative was dropped from further consideration. A photograph of the physical model during Phase A testing is presented in Figure 1.

Figure 1. 1:65 Scale Physical Model for Phase A Testing

Additional Physical Model Testing

After having received the data that was collected from the model, and having observational experience with the 1:65 scale model, several questions remained.

The District's Structural Design Section was not comfortable that sufficient data had been collected to be able to accurately analyze the stability of the stilling basin, both for existing conditions and for alternative analysis and design. The individual components of the basin, the baffle blocks, the space between the baffle blocks and the end sill at that scale, were too small to accommodate transducers for collection of pressure data. This was necessary to obtain a more accurate accounting of the forces that would be exerted on the stilling basin during large flood spillway discharges, such as for the PMF.

During the course of DDR study, a question was raised as to whether it would be necessary to provide protection for the exposed rock surface between the end sill of the “upper” stilling basin and the stilling weir. Determination of what protection measures, if any, that would be required was critical to the stability analysis and design of the spillway monoliths of the main dam. There was a concern that a strong horizontal eddy current, or roller, would develop as the flow stream exits the “upper” stilling basin, and the rock underneath and on the downstream side of the end sill would be scoured. Observation of the performance of the existing stilling features in the 1:65 scale model was inconclusive. The flow across the stilling basin and through the reach of exposed rock floor upstream of the stilling weir was too turbulent and the scale of the model was too small to allow collection of meaningful velocity data with respect to both magnitude and direction.

Again, CHL was consulted, and arrangements were made to conduct additional physical model studies. For this study effort, a 1:36 scale model was constructed to replicate a section through the spillway from the lake to the outlet channel downstream of the stilling weir. This model was constructed with a plexiglass wall along one side to facilitate observation of the flow beneath the surface of the water. The scale of this model was selected to conform to a similar sectional model, as was developed by the Carnegie Institute of Technology in 1937. That model was used in the original design of Bluestone Dam for the development of the features and dimensions of the stilling features.

The 1:36 scale sectional model provided for testing with a fixed bed and a movable bed between the stilling weir and the “upper” stilling basin. The fixed bed was used to collect pressure data for the baffle blocks and end sill of the upper stilling basin, the exposed rock surface, and the stilling weir. The movable bed was used to provide a qualitative assessment of the potential for scour and the propensity of the “upper” stilling basin end sill to be undermined.

Figure 2. 1:36 Scale Physical Model w/ Movable Bed

Conclusions

The results of the 1:65 scale model testing provided a more efficient discharge rating for both the spillway and the previously unused hydropower penstocks than had been computed for previous studies. In particular, the transition zone between weir flow and the beginning of pressure flow at the bottom of the spillway gates provided additional discharge capacity near the crest of the PMF pool. The spillway and penstock rating curves for earlier Evaluation Report studies and for the current DDR studies with the improved rating are shown on Figures 3 and 4.

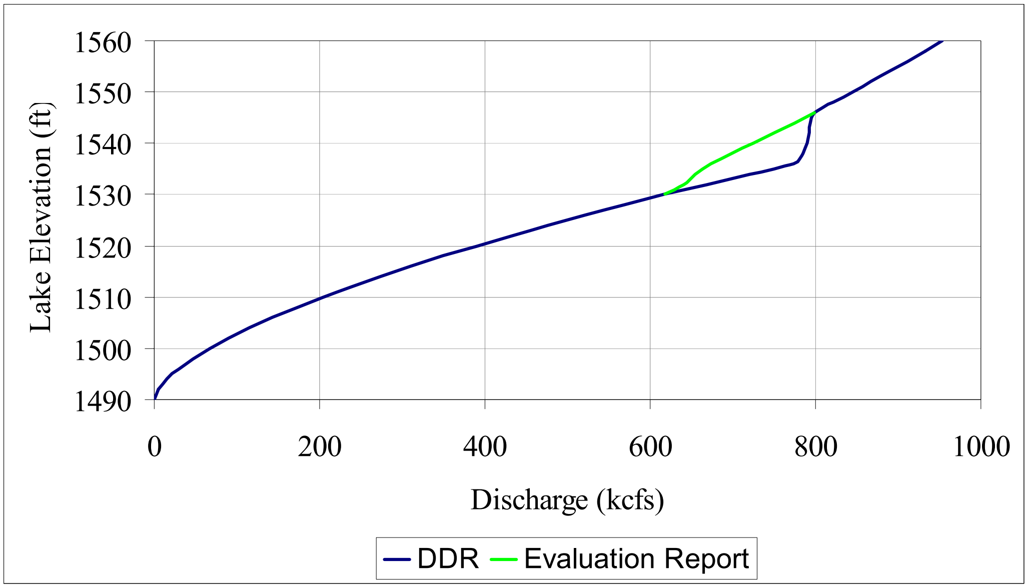

Figure 3. – Spillway Discharge Ratings

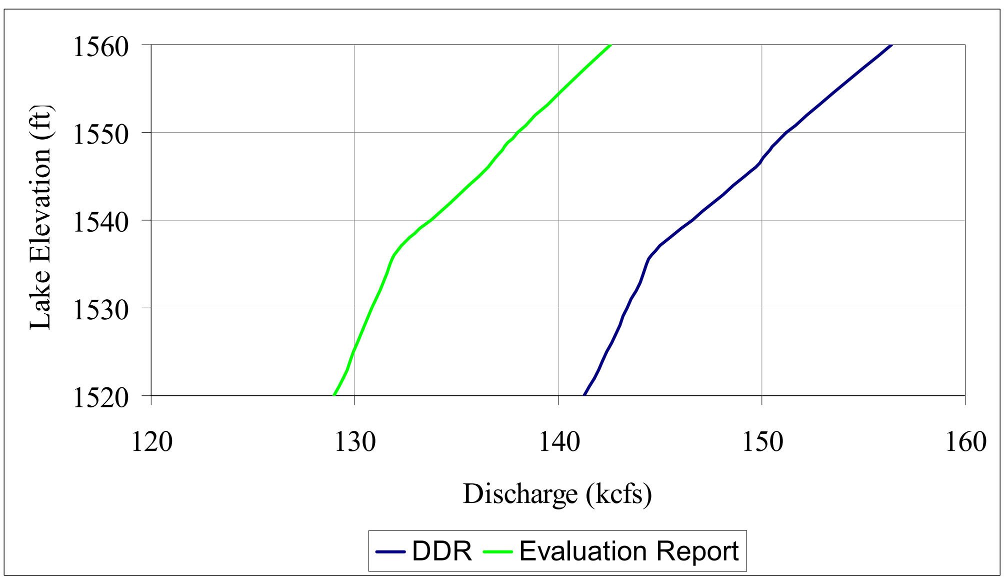

Figure 4. – Penstock Discharge Ratings

The use of the improved discharge rating curves resulted in a reduction of the crest elevation of the PMF pool from elevation 1546.8 to 1542.2. This in turn resulted in a savings of construction costs for correction of the hydrologic deficiency by reducing the height of the wall to be added to the top of the dam. Due to the reduction of the hydrostatic loading on the dam, an additional savings will be afforded by the reduction of anchoring requirements for the stability of the dam.

Other results of the 1:65 scale model studies included the indication that cavitation should not persist during the significant increase in spillway discharge over the original design condition. The lowest pressure recorded during at the surface of the ogee spillway crest was -14 feet. EM 1110-2-1603 indicates that -15 feet of negative pressure is the limit for low pressure on the spillway face. A nappe profile was obtained from the modeling effort for use in determining the required height of a stop log structure on each side of the spillway on the top of the dam to contain spillway flow with the confine of the spillway chute. This minimizes the opportunity for spillway flow to spread laterally along the top of the dam and contribute to erosion at the toe of the dam. Similarly, the height and length of training wall extensions at the bottom of the spillway chute were optimized, providing another cost savings. Releases from the penstocks produced a relatively high velocity eddy current along the toe of adjacent dam monoliths and the right descending bank. Flow deflectors at the ends of each penstock and an array of training wall lengths between the penstock outlet the adjacent dam monolith were tested. While the flow deflectors proved to be ineffective, testing provided a training wall length that significantly reduced the velocity of the eddy current, which in turn reduced the stone slope protection requirements for the toe of the dam monoliths and the adjacent stream bank. Pressure data was also collected for the upstream face of the spillway gates and piers for use in the structural stability analysis. Finally, pressure data was collected along the floor of the stilling pool for use in analyzing the stability of the rock surface between the end sill of the “upper” stilling basin and the stilling weir.

The 1:36 scale sectional modeling, particularly with the fixed bed, has provided additional pressure data for the region within the “upper” stilling basin for more detailed structural stability analysis of that feature. Also, pressure data has been collected for the upstream face, the ogee crest, and the stilling basin baffle blocks and end sill of the stilling weir for stability of that feature. A better approximation of the water surface in the stilling pool has also been provided for use in the stability of the existing training walls on either side of the stilling basin and weir. However, the additional pressure data that was collected within the “upper” stilling basin did not completely fill the void for an accurate determination of the loading for stability analysis purposes. Therefore, yet another modeling testing phase was conducted with the fixed bed in an effort to attain accurate loading parameters. The details of this spinoff will be presented in another paper by the modelers from CHL, and will be documented in the proceedings of the conference.

While the results of the 1:36 scale sectional modeling with the movable bed are considered to be qualitative in nature, the observation of the model’s performance has provided a basis for major decisions regarding the extent of remediation that will be provided for the stilling features. The results are considered to be qualitative because no assurance can be provided that any movable bed material in the model will perform, or behave in the same manner as the existing rock material during the occurrence of large flood releases, such as for the PMF. The existing rock is composed of layered and laminated orthoquartzite and carbonaceous shale. How and when these layers will begin to become unraveled and fail cannot be accurately replicated in the model. However, observation of the performance of the model indicates that a strong horizontal eddy current, or roller, does not develop at the end sill as the flow stream exits the "upper" stilling basin. At higher discharges, the flow appears to deflect slightly upward from the "upper" stilling basin, and follows a downstream trajectory similar in nature to a flip bucket spillway. This flow stream impacts the upstream face of the stilling weir and the rock surface at the bottom. As a result, a less violent horizontal roller current rotates underneath the trajectory of the high velocity jet, as if to support that jet. The bottom of the horizontal roller moves upstream toward the "upper" stilling basin, while the top moves downstream in synch with the higher velocity jet above it. As material is eroded from the vicinity of the upstream face of the stilling weir, most is carried over the weir and downstream. However, the horizontal roller transports part of the material upstream and deposits it near the end sill, resulting in an aggradation of material above the original surface. Meanwhile, prior to the crest of the PMF, the erosion pattern at the weir progresses to the point of failure of the weir and downstream portions of the training walls, as indicated in Figure 5. Releases for the crest of the PMF resulted in scour to the bottom of the model floor.

Figure 5. – Undermining of Stilling Weir with Lake Elevation 1530
(Top of Existing Dam = 1535, Crest of PMF = 1542.2)

The movable bed material was restored to initial conditions and releases were made to the condition where the stilling weir and adjacent training walls were believed to fail. The model was then dewatered, the weir was removed, and discharge was increased to the PMF level. This resulted in a more elongated and shallower scour pattern, with some aggradation continuing to occur at the "upper" stilling basin end sill. Finally, the end sill of the "upper" stilling basin was removed, in addition to absence of the stilling weir, and releases were made. These conditions resulted in the loss of material at the end of the end sill. However, at the end of the PMF release, the associated degradation did not exceed the bottom of the existing concrete underneath the end sill.

Based on the observations of the movable bed modeling, the District has decided to provide anchoring for the “upper” stilling basin and its baffle blocks and end sill to prevent horizontal sliding. Due to the apparent stability at the end of the end sill, a new and deeper key wall will not be provided to prevent undermining, as was originally thought would be necessary. Neither will any additional stability measures be provided for the existing spillway training walls and stilling weir. Due to the relatively infrequent occurrence of major floods, such as the PMF, and in conjunction with the apparent stability at the end sill, this is believed to be a reasonable risk to assume because the integrity of the dam is maintained. This decision is also made with the expectation that the failure of the lower ends of the training walls and stilling weir can be remediated prior to the occurrence of a successive major storm event.

In conclusion, the physical modeling of Bluestone Dam and its spillway component has provided key information and data for the detailed design that could not otherwise be obtained or computed with a known degree of accuracy or confidence. However, the movable bed modeling and the ensuing use of its results may be debated for some time into the future. Nevertheless, the observation of that model’s performance in combination with the use of engineering judgment and the consensus of the quality assurance process will provide Bluestone Dam with the capability to function safely and successfully according to modern day design standards.

References

US Army Engineer Office, Huntington, WV, February 1937, New River – Bluestone Reservoir Project, Final Report – Laboratory Tests on Hydraulic Models of Bluestone Dam, Carnegie Institute of Technology, Pittsburgh, PA

US Army Corps of Engineers, Huntington District, December 1995, Reconnaissance Level Evaluation Report for Bluestone Lake Hydropower Study (Draft)

US Army Corps of Engineers, Huntington District, December 1996, Bluestone Drift and Debris Evaluation Study - Report and Environmental Assessment/FONSI

US Army Corps of Engineers, Huntington District, 1998, Bluestone Lake Dam Safety Assurance Program Evaluation Report

Fuller, Billy D., Coastal and Hydraulics Laboratory, March 2002, Bluestone Lake Dam, West Virginia –Rating Curve and Overtopping Study, ERDC/CHL TR-02-2

US Army Corps of Engineers, Huntington District, January 2002, Design Documentation Report (DDR) DSA Phase II, Bluestone Lake, WV (50% ITR Review Copy)